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DESIGN, CONSTRUCTION AND COST OF ROCK CHECK DAMS

by Burchard H. Heede

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DESIGN, CONSTRUCTION, AND COST
OF
ROCK CHECK DAMS

by

Burchard H. Heede, Hydraulic Engineer

Rocky Mountain Forest and Range Experiment Station ¹

¹ Central headquarters maintained in cooperation with Colorado State University at Fort Collins.

CONTENTS

	Page
Study area	2
Methods	3
Types of rock check dams applied	4
General design criteria	7
Construction	17
Costs	21
Results and recommendations	23
Literature cited	24

Design, Construction, and Cost of Rock Check Dams²

by

Burchard H. Heede

Check dams built from loose rock have been used to control the flow of water in gullies and canals since ancient times. Rock remained the principal construction material during the last large-scale installation of check dams in the 1930's when the Civilian Conservation Corps rehabilitated depleted lands. After this period, steel sheets and concrete were the primary materials used in check dams. Although costs of steel or concrete check dams are much higher than those built from loose rock, the use of rock was neglected.

Perfection of modern construction materials was not the only reason for the neglect of rock. Rock check dams, as conventionally constructed, require large amounts of hand labor. New construction methods were not developed to save manual work by application of modern equipment. Also, older rock check dams failed frequently because they were often inadequately designed, placed at unsuitable locations, or not maintained.

The failure to use rock check dams and to improve their design was not justified. Based on studies of 25-year-old structures, Heede (1960)³ demonstrated that rock check dams can effectively control gullies, and that these structures are long lasting if the conditions of flow and channel are considered in their design and placement. Figures 1 and 2 show a successful rock check dam constructed during

our present study. That rock check dams can be effective for more than 40 years is demonstrated by structures in California (fig. 3).

To learn more about the feasibility of using modern techniques in the design and construction of rock check dams, a study was set up with the following objectives:

1. Design and install four different types of rock check dams: double-fence, single-fence, wire-bound, and rock only.
2. Design and construct gully head-cut controls consisting of rock only.
3. Test the feasibility of using conventional construction equipment in the installation of the structures.
4. Determine the costs of the different types of structures and their cost relationships.

²Research reported here was conducted in cooperation with the White River National Forest, Glenwood Springs, Colorado, and Region 2 Office of the U. S. Forest Service, Denver.

³Names and dates in parentheses refer to Literature Cited, p. 24.

Figure 1.--A double-fence rock check dam during the first heavy flow of spring snowmelt. Suspended-sediment concentrations of 35,000 p.p.m. were sampled below this dam.



Study Area

The structures to be tested were installed on the upper Alkali Creek watershed, tributary to the Colorado River in Colorado. The watershed, about 1 square mile in size, is located on the Rifle Ranger District of the White River National Forest, about 20 miles south of the town of Silt. The elevation ranges from 7,500 to about 8,500 feet. Marine shales and sandstones of the Wasatch formation provide the main parent materials for the soils. Silt and clay contents of the soils are high, about 38 and 50 percent, respectively. The vegetation consists mainly of Gambel oak, big sagebrush, Kentucky bluegrass, and western wheatgrass. Sagebrush grows predominantly on the bottom lands and the south slopes. Gambel oak occupies ridges and the north slopes.

Numerous gullies of varying sizes dissect the bottom and the side slopes of the watershed. Gully flow is ephemeral, and occurs only during spring snowmelt and at times of exceptionally intense summer storms. The load of these flows consists mainly of fines and sand; gravel and boulders are only occasionally transported. A suspended-sediment concentration of 35,700 p.p.m. was sampled at a discharge of 7 c.f.s. during the snowmelt season. This concentration corresponds to a sediment discharge of 16 pounds per second, or about 10 cubic feet per minute.



Figure 2.--The dam of figure 1 after the snowmelt season. The catchment basin of the dam is filled with sediment to the crest of the spillway, and vegetation is invading the deposits.



Figure 3.--Looking upstream on two loose rock check dams, representing hand-placed dry masonry walls, that controlled this tributary gully to Haines Canyon, San Gabriel Mountains, very effectively for more than 40 years. Grass formed thick sod covers on the sediment deposits, adding stability to the structures and the gully.

Records from 1961 through 1965 indicate that the area received an average total precipitation per water year (October 1 to September 30) of 18.57 inches. Judged on the basis of longer records from weather stations, and considering the elevation difference to these stations, this amount appears to be somewhat above normal. Of the total precipitation, 57 percent was snow and 43 percent rain. As illustrated below, several cloudburst storms hit the study area during the period of record:

	Maximum intensity		Total
	(Inches/hr)	(Minutes)	precipitation (Inches)
1963:			
7/9	1.28	15	0.51
7/15	1.80	3	.19
7/23	1.66	21	.60
9/20	1.50	12	.42
1964:			
7/30	1.00	9	.27
8/12	1.90	6	.54
8/12,13	1.20	5	.74
8/27	3.00	1	.38
1965:			
7/31	.96	5	.28
9/2	2.20	3	.29
9/2	3.00	2	.10
9/5	4.20	1	.33
9/11	1.50	10	.47
9/29	4.80	1	.61

The structures did not receive the most severe test during the flows from cloudbursts, but during flows from snowmelt because the melt flows lasted over long periods of time. In the spring of 1964, the melt water ran for 7 weeks continuously in the gullies, and the water saturated gully side slopes and bottoms as well as the foundations of the dams. Furthermore, since recession flows of prolonged melt seasons decrease their load with time, as observed on the study area,⁴ check dams are endangered by the increased capability of the water to cause erosion at the structural sites.

⁴Heede, Burchard H. *Suspended sediment of gully flows related to duration of flow*. 4 pp. 1964. (Unpublished office report on file at Rocky Mountain Forest and Range Exp. Sta., Ft. Collins, Colo.)

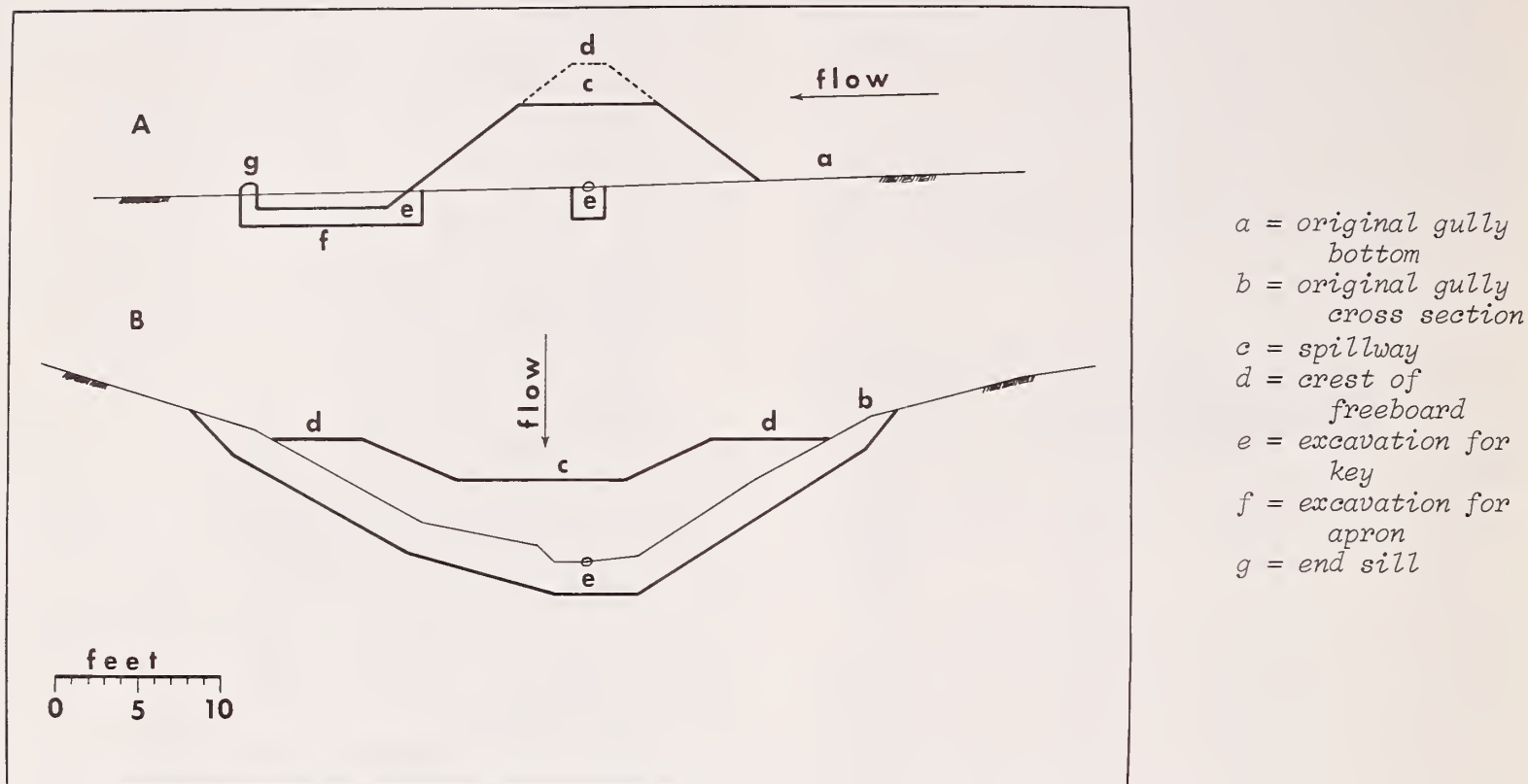
A check dam that is either too small or too large to achieve its objective results in a waste of funds. In a strict sense, knowledge basic to the proper design of check dams is not yet adequate to assure that they are neither over nor under designed with respect to intrinsic stability or gully control. It is apparent, however, that available knowledge can be effectively applied only when the relevant features of a prospective dam site have been fully evaluated. Careful surveys of sites are therefore prerequisite to dam design.

The gullies of the study area were surveyed with tape, survey rod, Abney hand level, and staff compass. During the survey, reference points were established alongside the gullies by use of transit and engineer's level to facilitate future resurveys. Channel gradients and alinements, bottom and upper widths, and depths of gullies were determined and drawings made of the longitudinal profiles, cross sections, and plan views. A radial-line plotter applied to aerial photographs facilitated computations of areas.

Nine gullies were randomly selected for the installation of the structures. Gully geometry and length varied greatly. Thus, the width ranged from 5 to 50 feet, the depth from 2 to 30 feet, the length from 50 to 4,400 feet, and the gradient from 3 to 18 percent.

Based on the field survey, 122 check dams and 12 head-cut controls were designed in the office. Locations of the dams were determined by the spacing requirements. Since information on gully cross sections was available only for survey stations spaced about 50 feet, it was necessary to check on the suitability of those located between the stations, and to obtain the missing survey data. The suitability of a site was evaluated by factors such as access, soil piping, or extremely wide cross section relative to neighboring locations. If a site was found unsuitable, another was chosen. In some cases, small changes in location of a dam to take advantage of a rock outcrop or a narrow channel section resulted in substantial cost savings. The spacing between the dams was checked and if the requirements were exceeded, additional dams were designed.

Figure 4a.--Construction plans for a loose-rock check dam.
 A. Section of the dam parallel to the centerline of the gully.
 B. Section of the dam at the cross section of the gully.



It was desired to have equal numbers of different types of check dams. Yet, no type was assigned where it would fail.

During the field season following the period of design work, a contractor installed the structures according to the plans and under direct supervision. Motorized equipment was employed to the fullest extent possible. The rock to be used in the dams was quarried on the study area; all other construction materials were purchased on the open market.

Types of Rock Check Dams Applied

Loose-Rock Check Dams

The design of loose-rock check dams is illustrated by the plottings of the cross section of the structure and of the elevation parallel to the center line of the gully (fig. 4a). These drawings were used for the calculation of volumes of excavation and of rocks required in the construction, and later served in the field as construction plans.

Loose-rock check dams consist of rock only (fig. 4b). Since this type of structure is not reinforced, the angle of rest of the rock should determine the slopes of the dam sides. This angle depends on the type of rock, the weight, size, and shape of the individual rocks, and the size distribution among all rocks. If the dam sides are constructed at an angle steeper than that of rest, the structure will be unstable and may lose its shape during the first heavy runoff. For the design of check dams, the following rule of thumb can be used: the angle of rest for angular rock corresponds to a slope ratio of 1.25 to 1.00; for round rock, 1.50 to 1.00.

Wire-Bound Rock Check Dams

A wire-bound check dam is identical in shape to that of a loose-rock dam. It differs from the latter in that the loose rock is enclosed in wire mesh (fig. 5). The objective of the mesh is to reinforce the structure. The tension in the mesh stresses the rock and thus causes tighter anchorage of the individual

Figure 4b.--Upstream view of a loose rock check dam during first runoff from spring snowmelt after construction. Heavy bouldery material scattered in this gully was collected and used together with quarry rock in the structure. The effective height of the dam is about 4 feet. Approximately 15 c.f.s. of water and sediment are discharging over the spillway.

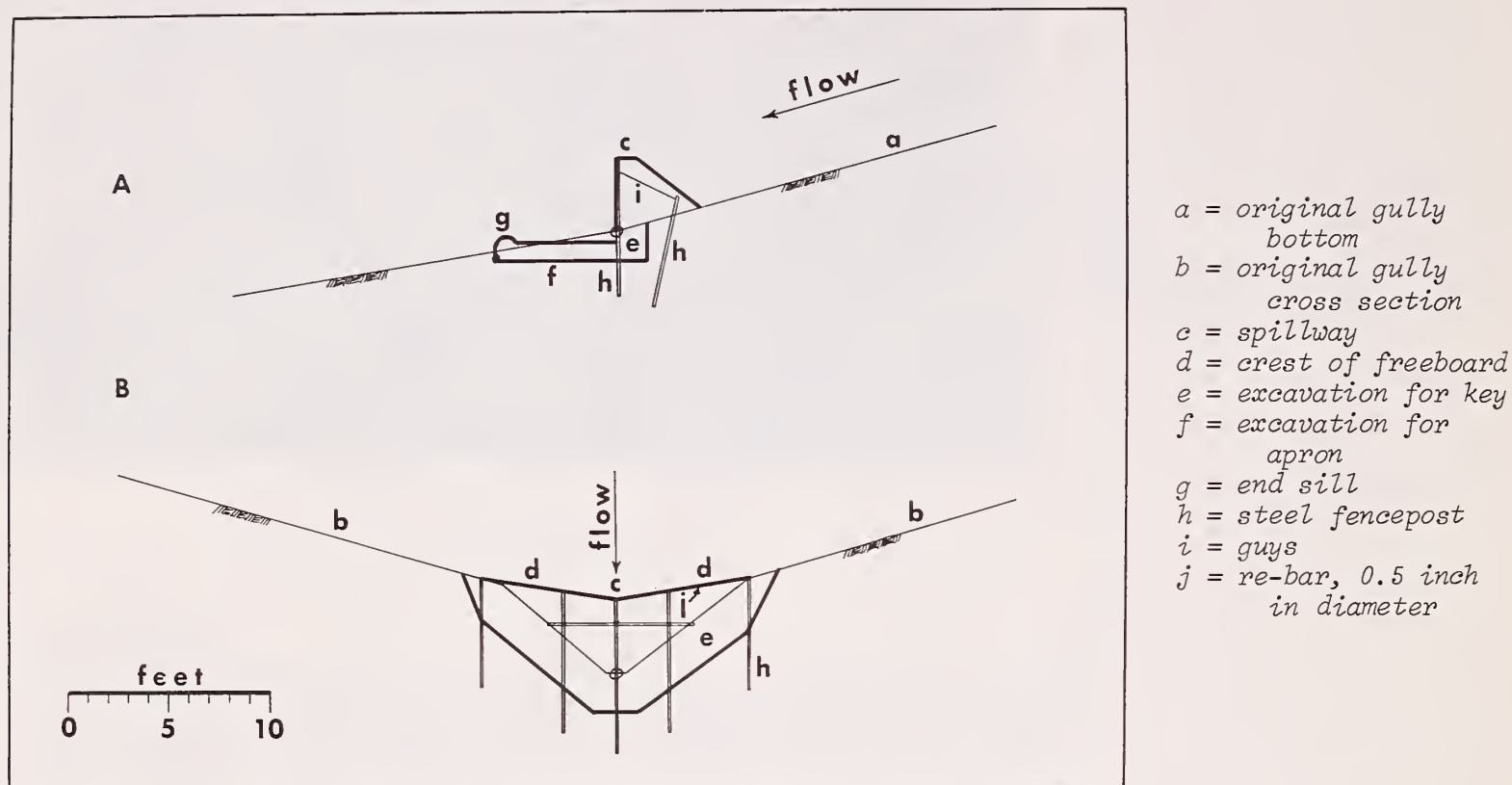


Figure 5.--This wire-bound check dam was built from selected river rock because of temporary difficulties at the rock quarry. Some sediment collected on the apron (lower left) during the first flow from spring melt after construction. The peak flow of the snowmelt waters reached a depth of 0.75 foot in this gully, where the bottom was about 2.5 feet wide. Only a small amount of water overtopped the spillway. Most of the flow percolated through the dam. Effective dam height is 4.7 feet.



Figure 6a.--The construction plans for the single-fence rock check dam shown in figure 5b.

- A. Section of the dam parallel to the centerline of the gully.
B. Section of the dam at the cross section of the gully.



pieces. The flexibility within the wire mesh is sufficient to permit adjustments in the structural shape, if the dam sides are not initially sloped to the angle of rest. Therefore, the same rock design criteria are required for a wire-bound dam as for a loose-rock structure.

The wire mesh should: (1) be resistant to corrosion, (2) be of sufficient strength to withstand the pressure exerted by flow and rocks, and (3) have openings not larger than the average rock size in the dam.

Single-Fence Check Dams

Single-fence rock check dams (fig. 6) differ greatly in shape and requirements of construction materials from the loose-rock and wire-bound dams. These structures consist of a wire-mesh fence, fastened to steel posts and strung at right angles across the gully, and a loose-rock fill, piled from upstream against the fence. The rock fill can be constructed at an angle steeper than that of rest because of two reasons:

1. The impact of future flows will have a tendency to push the individual rock into the fill and against the dam.
2. Expected sediment deposits will add stability to the fill and will eventually cover it.

In the design of this type of check dam, emphasis should be placed on the specifications for the wire mesh, and the setting, spacing, and securing of the steel posts. The wire mesh specifications will be the same as those for the wire-bound dams.

The steel posts should be sufficiently strong to resist the pressure of the rock fill and the flows. They must be driven into the gully bottom and side slopes to a depth that insures their stability in saturated soil. If it is impractical to drive posts to sufficient depths, the stability of the posts should be enhanced by guys. These guys should be anchored to other posts that will be covered and thus held in place by the rock fill.

In general, the fenceposts of the study dams were spaced no more than 4 feet to prevent

Figure 6b.--The crest of the spillway of this single-fence rock check dam is 3.7 feet above the original gully bottom. The rod, 5.5 feet long, was set below the dam on the apron. A backhoe placed the rock into this structure. Note the gully side-slope protection below the dam.



excessive pouching of the wire mesh. Where a maximum spacing of 5 feet was applied, the fence was reinforced by a steel post fastened horizontally between the vertical posts. Excessive pouching of the wire mesh reduces the structural height and impairs the stability of the dam.

Double-Fence Rock Check Dam

The main features of double-fence rock check dams are two wire-mesh fences, strung across the channel 2 feet apart (fig. 7). The space between the fences is filled with loose rock. Design specifications for the wire mesh and the steel posts are the same as for single-fence structures. To counteract the pressure exerted by the rock fill against both fences, guys connect opposite fenceposts.

Loose Rock Controls for Gully Head Cuts

The control of head cuts to stop headward extension of gullies is an important feature in gully treatment. Figure 8a depicts a section of the control work, plotted parallel to the center line of the gully. As in loose-rock check dams, the quality, size, shape, and size distribution of the rock are of special importance to the success of the structure, and therefore require prime consideration in the

design. To make the placement of the rock against the wall of the head cut possible, this wall must be sloped back.

If the toe of the rock fill should give way to the forces of the expected flow, the fill would be lost. Therefore, the stabilization of this toe received emphasis in the design. A loose-rock dam was designed to act as an energy dissipator for the chuting flows and as a catch for sediment (fig. 8b). It was expected that the sediment depositions would further stabilize the toe of the rock fill by encouraging vegetation during periods without channel flows.

General Design Criteria

Rock

The quality, size, and size distribution of the rock used in construction of a check dam exert great influence on the success and life-span of the structure. It is obvious that rock that disintegrates rapidly when exposed to water and atmosphere will lead to a short structural life. Further, if only small rocks are used in a dam, they may be moved by the impact of the first large water flow, and the dam quickly destroyed. In contrast, a check dam constructed of large rocks that leave large voids in a structure will offer resistance to the flow, but may create water jets through

Figure 7a.--The construction plans for the double-fence rock check dam shown in figure 6b.

- A. Section of the dam parallel to the centerline of the gully.
B. Section of the dam at the cross section of the gully.

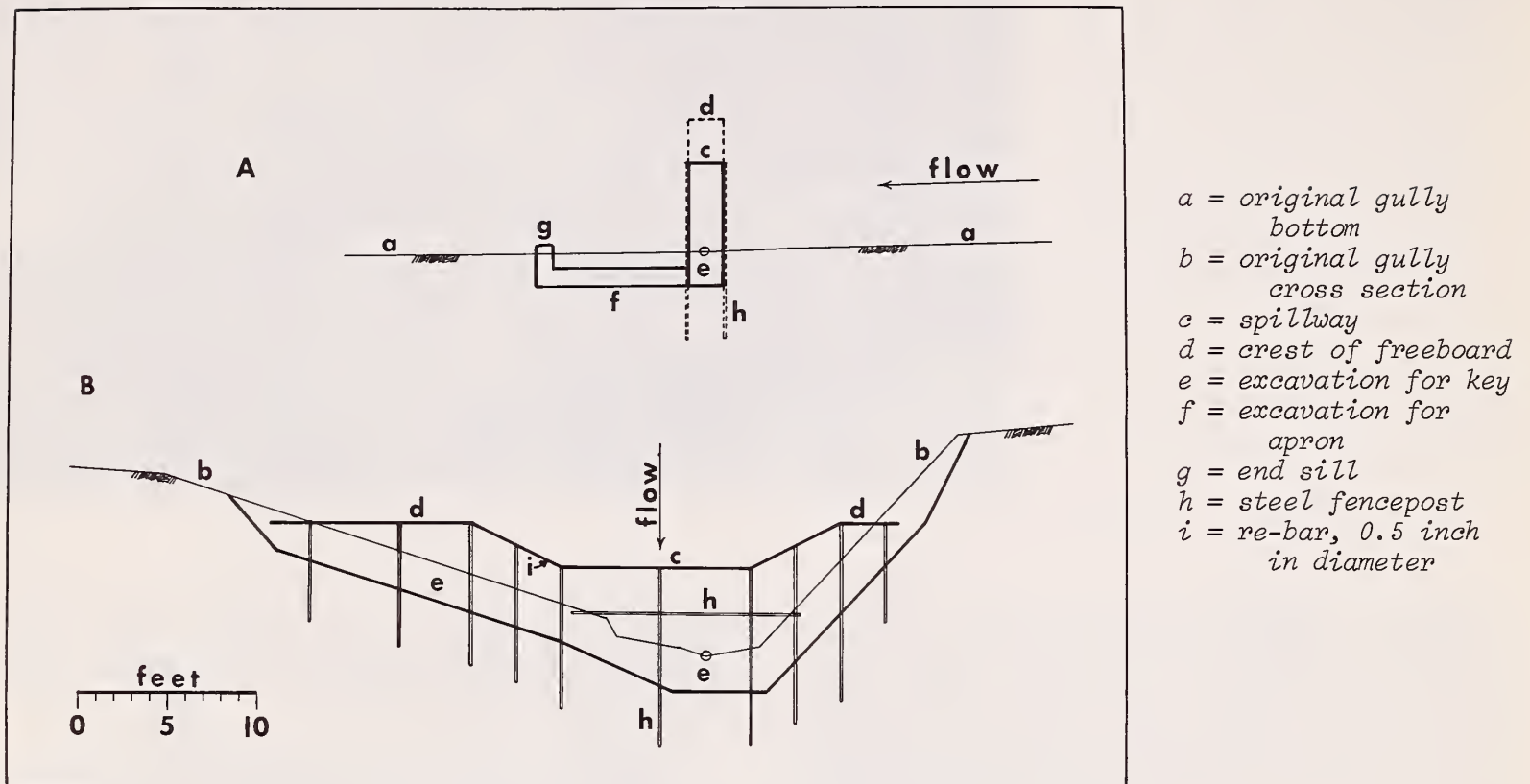


Figure 7b.--Looking upstream at a double-fence rock check dam with an effective height of 5.5 feet. Water is still standing on the apron. A small flow developed from a light storm 1 day before this photo was taken. Dam construction was finished only 1 day before the storm.



Figure 8a.--The construction plan of the gully head-cut control with a loose-rock check dam as shown in figure 7b. The section of the structures is illustrated parallel to the centerline of the gully.

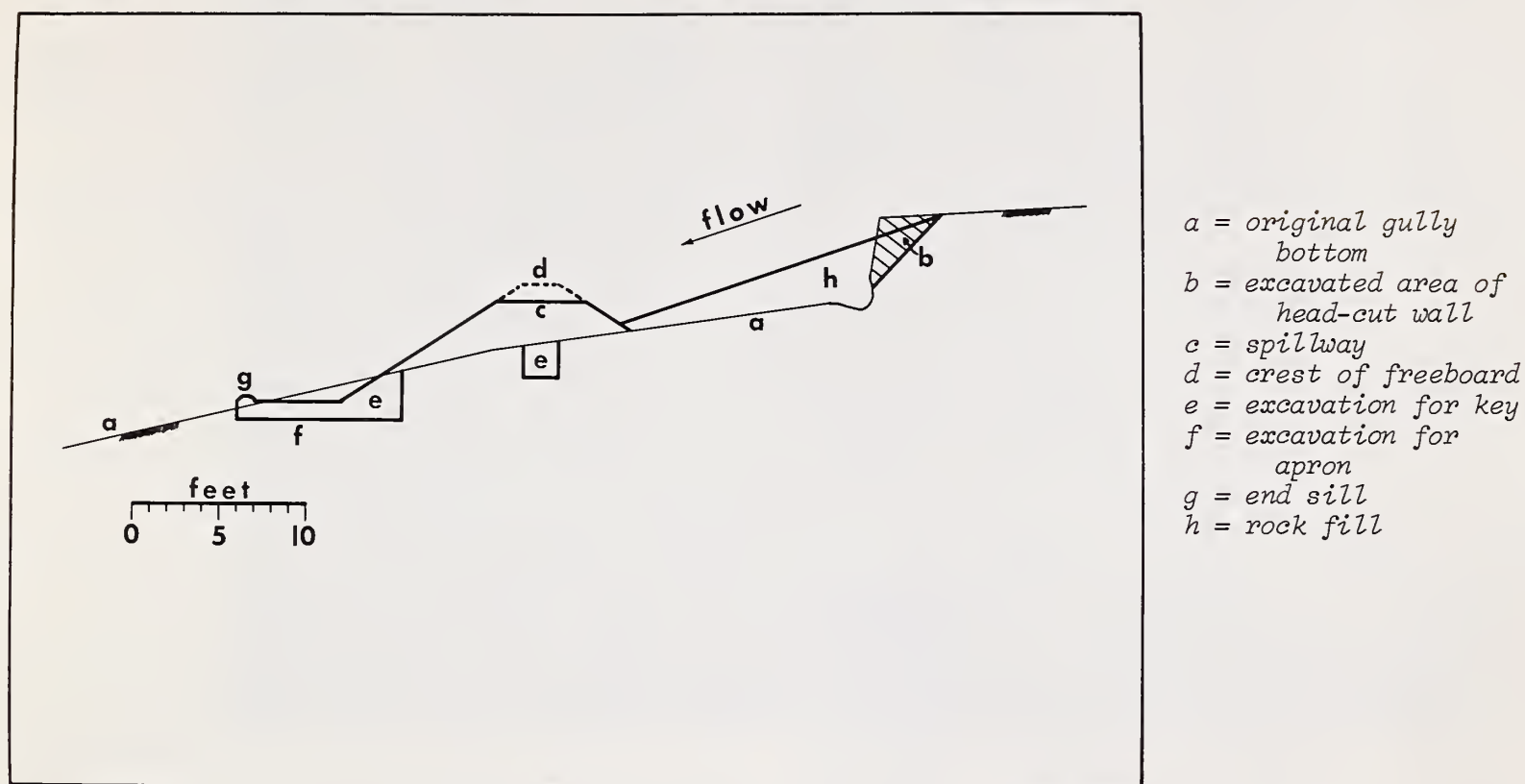


Figure 8b.--Looking upstream on a loose-rock check dam installed at the toe of a rock fill of a former gully head cut in July 1961. The rod on the apron of the check dam is 5.5 feet long. The depth of the former head cut was 5.8 feet. The photo, taken on September 2, 1964, indicates the stabilization of channel and head cut. This gully experienced several spring melt flow seasons and some small flows from summer storms. Little sediment was deposited above the dam.



the voids (fig. 9). These jets can be highly destructive if directed toward openings in the bank protection work or other unprotected parts of the channel. Large voids in check dams also prevent the accumulation of sediment above the structures (fig. 10). In general, this accumulation is desirable because it increases the stability of structures and enhances stabilization of the gully.

Large voids will be avoided if the rock is well graded. Well-graded rock will permit some flow through the structure, which will release part of the hydrostatic pressure against the dam. The majority of the rock should be large enough to resist the flow (fig. 11).

The designs were based on quarry rock, consisting of medium to hard sandstone, with diameters ranging from 0.3 to 1.0 foot. Where only loose rock was to be used in the dams, the design permitted the use of bouldery material with a diameter as large as 3 feet. The boulders occurred as a natural scattering around and within the gullied area.

Figure 9.--When subjected to its first heavy flows during the period of spring snowmelt, a rock check dam releases some of the hydrostatic pressure of the water through the pores of the structure.



Figure 10.--Downstream view. This double-fence rock check dam had accumulated sediment 1.5 feet deep during the peak flow of the spring melt season, but the almost clear recession flows carried most of the deposits through the dam. Insufficient grading of the rock had resulted in too many large rocks and great pores in the structure. A remnant of the former deposits is visible on the right side of the gully bottom.



Figure 11.--Downstream view of a successful double-fence rock check dam at the end of the spring melt following construction. The structure accumulated sediment to a depth of 1.5 feet. The last recession flow of the season is running over the deposits and through the dam. The pipes on the gully bottom and the side slope are crest gages.



Spacing

The location of a check dam will be determined primarily from the required spacing of the structures. Requirements for spacing depend on the gradients of the sediment deposits expected to accumulate above the dams, the effective heights of the dams, the available funds, and the objective of the gully treatment. If, for instance, the objective is to achieve the greatest possible deposition of sediment, widely spaced, high dams would be constructed. On the other hand, if the objective is mainly to stabilize the gully gradient, the spacing would be relatively close and the dams low.

In general, the most efficient and most economical spacing is obtained if a check dam is placed at the upstream toe of the final sediment deposits of the next dam downstream.

Figure 12.--These dams were built from loose rock only, and were spaced to allow full utilization of the catchment capacity of structures. At each check dam, the turbulent flow is transformed into more tranquil flows. Thus, if several dams are spaced in a gully, the regimen of the flows will be changed to benefit stabilization processes.

This ideal spacing can only be estimated, of course, to obtain guidelines for construction plans.

The long-range objectives of the study reported here require spacings of check dams great enough to allow the full utilization of the sediment-holding capacity of the structures (fig. 12). To determine this spacing requires definite knowledge of the relationship between the original gradient of gully channels and that of sediment deposits above check dams placed in these gullies. This relationship has been hypothesized by several authors (Ferrel 1959, Heede 1960).⁵ Woolhiser and Lenz (1965) demonstrated that not only the original channel gradient influences the deposition slope, but also the width of the channel at the structure and the height of crest of spillway above the

⁵Kaetz, A. George, and Rich, Lowell R. Report of surveys made to determine grade of deposition above silt and gravel barriers. (Unpublished office report of U. S. Soil Conserv. Serv., Albuquerque, N. Mex., Dec. 5, 1939. Abstracted in bibliography of unpublished sedimentation data and annotated bibliography of upstream effects of dam construction on stream regimen, Sedimentation Subcommittee of Pacific Southwest Interagency Committee, p. 8, March 1961.)



original channel bottom. Because relationships developed so far have been entirely empirical, further research is necessary to establish the theoretical basis.

Deposits above earth check dams existing on the study area were examined for guidance in the spacing of dams constructed under this study. Based on data so obtained, it was assumed that, in gullies of less than 20 percent gradient, the dams would not interfere with sediment catch if their spacing was predicted on the slope of the expected deposits being 0.7 of the original gully gradient. Thus, for a gully gradient of 15 percent, the gradient of the expected deposits was assumed to be 0.7×15 , or 10.5 percent. Then for 6-foot-high dams, the spacing between dams would be $6/0.105$ or approximately 57 feet.

For gully gradients exceeding 20 percent, it was assumed that expected deposits of sediment would have a gradient of 0.5 that of the gully. Thus, for a gully gradient of 24 percent, the sediment gradient would have a slope of 12 percent and the spacing between 6-foot-high dams would be $6/0.12$ or 50 feet.

Keys

The keying of a check dam into the side slopes and the bottom of the gully greatly enhances the stability of the structure. Such keying is important in gullies where expected peak flow is large, and where highly erosive soils such as soils with high sand content exist. Successful loose rock check dams without keys were installed in soils derived from Pikes Peak granite and did not experience an estimated peak larger than 8 c.f.s. (Heede 1960).

The objective of the key located in the gully side slopes is to prevent destructive flows of water around the dam and consequent scouring of the banks. Scouring could lead to gaps between dam and bank that would render the structure ineffective.

The keys minimize the danger of scouring and tunneling around check dams because the route of seepage is considerably lengthened. As voids in the key become plugged, the

length of the seepage route increases. This increase causes a decrease in the flow velocity of the seepage water and, in turn, a decrease of the erosion energy.

The part of the key placed into the gully bottom is designed to safeguard the check dam against undercutting at the downstream side. Therefore, the base of the key, which constitutes the footing of the dam, must be designed to be below the surface of the apron. This is of particular importance for fence-type structures because of the greater danger of scouring at the foot of these dams. The water flowing over the spillway forms a chute that creates a main critical area of impact where the hydraulic jump strikes the gully bottom. This location is away from the structure. The sides of loose-rock and wire-bound check dams slope onto the apron, on the other hand, and no freefall of water occurs.

The design of the keys called for a trench, usually 2 feet deep and wide, dug across the channel at the construction site (fig. 13). Where excessive instability was demonstrated by large amounts of loose materials on the lower part of the channel side slopes or by large cracks and fissures in the bank walls, the depth of the trench was increased to 3 or 4 feet.

Dam construction started with the filling of the key with loose rock. Then the dam was erected on the rock fill. It became apparent after the first channel flows following construction that rock size distribution in the keys should be watched carefully. In some keys, smaller rock sizes were missing, voids in the keys were large, and velocities of flow within the key led to washouts of the bank materials. Since the rock of the keys is embedded in the trench and therefore cannot be easily moved, it is advantageous to use smaller materials, such as a mixture with 80 percent smaller than 3 inches. This mixture seems to be adequate for the stability of the keys by present knowledge.

Height

The effective height of a check dam is the elevation of the crest of the spillway above the original gully bottom (fig. 14). The height not



Figure 13.--

This backhoe excavated the keys for check dams where gully cross sections were small and the construction sites easily accessible.



Figure 14.--

The effective height of check dams influences not only the volume of future depositions, but also the spacing of structures. These double-fence rock check dams were installed on a gradient of 12.5 percent.

only influences structural spacing but also volume of sediment deposits. If the objective of gully control is to deposit large volumes of sediment, higher dams are superior to lower ones. Since the volume of sediment deposited varies approximately as the square of the effective height of the dam, a 6-foot structure accumulates about 9 times as much sediment as a 2-foot dam.

The rule of thumb for spacing dams, previously given and illustrated, may be used to determine, for a given gully gradient, the ratio of low dams to high dams required for efficient gully control. In a gully of 5 percent gradient, for instance, the ratio of 2-foot to 6-foot dams is 2; in a gully of 15 percent gradient, the ratio increases to 2.5. Yet in the first instance the 6-foot dams will cause the deposition of 4.5 times the sediment stored by twice as many lower dams; with a 15 percent gradient, the higher dams will accumulate 3.6 times more sediment.

In most cases, however, dam height will be restricted by one or all of the following criteria: (1) costs, (2) stability, and (3) channel geometry in relation to spillway requirements.

Cost criteria are discussed later. Stability of rock check dams cannot be calculated without detailed experiments in field and laboratory because of unknowns such as the porosity of a structure.

A maximum height of 7 feet was set for loose-rock and wire-bound dams, and 6 feet for single- and double-fence dams. All structures were severely tested during the first snowmelt season following construction, but none was destroyed.

In gullies with small widths and depths but large magnitudes of flow, the effective height of dams may be greatly restricted by the spillway requirements. This restriction may result from the spillway depth necessary to accommodate expected debris-laden flows.

Spillway

Since spillways of rock check dams may be considered broad-crested weirs, the discharge formula for that type of weir is applicable:

$$Q = CLH^{3/2}$$

where

Q = discharge in c.f.s.,

C = coefficient of the weir,

L = effective length of the weir, and

H = head of flow above the weir crest.

The value of \underline{C} varies between 2.6 and 3.6 over ranges of weir breadths from 0.5 to 15 feet and flow depths from 0.2 to 5.5 feet (King 1954). Because the exact value of \underline{C} depends on the roughness as well as the breadth and shape of the weir and the depth of flow, it cannot be readily determined for the spillway of a rock check dam. Therefore, land managers conventionally apply a mean value of 3. This value appears reasonable in the light of other inaccuracies that are introduced in calculating the design storm and its expected peak flow. For this reason also, the discharge calculations would not be significantly improved if they were corrected for the velocity of approach existing above a dam. Such a correction would amount to an increase of 5 percent of the calculated discharge at a head of flow of 2 feet over a dam 2.5 feet high, or 8 percent if the flow had a 3-foot head.

For structural gully control, design storms should be of 25 years' magnitude, and, as a minimum, spillways should accommodate the expected peak flow from such a storm. In mountainous watersheds, however, where forests and brushlands often contribute large amounts of debris to the flow, the size and the shape of spillways should be determined by this expected organic material. As a result, required spillway sizes will be much larger than if the flow could be considered alone. Spillways designed with great lengths relative to their depths are very important here. Yet, spillway length can be extended only within limits because a sufficient contraction of the flow over the spillway is needed to form larger depths of flows to float larger loads over the crest. The obstruction of a spillway by debris is undesirable since it may cause the flow to overtop the freeboard of the check dam and lead to its destruction.

The characteristics of the sides of a spillway are also important for the release of debris over the structure. Spillways with perpendicular sides will retain debris much

easier than those with sloping sides; in other words trapezoidal cross sections are preferable to rectangular ones. A trapezoidal shape introduces another benefit by increasing the effective length of the spillway with increasing magnitudes of flow.

The length of the spillway relative to the width of the gully bottom is important for the protection of the channel and the structure. Normally, it is desirable to design spillways with a length not greater than the available gully bottom width so that the waterfall from the dam will strike the gully bottom. There, due to the stilling-basin effects of the dam apron, the turbulence of the flow is better controlled than if the water first strikes against the banks. Splashing of water against the channel side slopes should be kept at minimum to prevent new erosion. Spillway length may have to exceed gully-bottom width in gullies with V-shaped cross sections, or where large flows of water and debris are expected relative to the available bottom width. In such cases intensive protection of the gully side slopes below the structures is required.

Apron

Stability aspects require the installation of aprons on the gully bottom and protective works on the gully side slopes below the check dams. Without protection, flows may easily undercut the structures from downstream and destroy them.

Aprons and bank protection works were built mainly from loose rock. To simplify the design procedures, a rule of thumb was adopted: the length of the apron should be 1.5 times the height of the structure in channels where the gradient does not exceed 15 percent, and 1.75 times where the gradient is steeper than 15 percent. The resulting apron lengths included a sufficient margin of safety to prevent the waterfall from hitting the unprotected gully bottom. The design provided for embedding the apron into the channel floor so that its surface would be roughly level and about 0.5 foot below the original bottom elevation.

At the downstream end of the apron, a loose rock sill was designed 0.5 foot high,

measured from channel bottom elevation to the crest of the sill. This end sill creates a pool in which the water will cushion the impact of the waterfall.

The installation of an end sill provides another benefit for the structure. Generally, aprons are endangered by the so-called ground roller that develops where the hydraulic jump of the water hits the gully bottom. These vertical ground rollers of the flow rotate upstream, and where they strike the gully floor, scouring takes place. Thus, if the hydraulic jump is close to the apron, the ground roller may undermine the apron and destroy it. The end sill will shift the hydraulic jump farther downstream, and with it the dangerous ground roller. The higher the end sill, the farther downstream the jump will occur. Since data on sediment and flow were not available, a uniform height of sill was used for all structures.

Gullies with ephemeral water carry frequent flows of small magnitudes. Therefore, it is advisable not to raise the crest of the end sills more than 0.5 to 0.75 foot above the gully bottom. End sills, if not submerged by the water, are dams and create waterfalls that may scour the ground below the sill. Usually, at higher flows some tailwater exists below a sill and cushions to some extent the impact from the waterfall over the sill.

Where the downstream nature of the gully is such that appreciable depth of tailwater is expected, the installation of end sills is not critically important. The hydraulic jump will strike the water surface and ground rollers will be weak.

Side-Slope Protection

Investigations have shown (Heede 1960) that check dams may be destroyed if flows scour the gully side slopes below the structures and produce a gap between the dam and the bank. Since intensive turbulence exists in the water below a check dam, eddies develop that flow upstream along each gully side slope. These eddies are the cutting forces. If it were practical, these currents could be prevented by the installation of long sills on the apron (Morris and Johnson 1943, U.S. Bureau of Reclamation

1960). These sills would separate the areas of fast flow from those of low velocities at each side of the stream. Gully channels are usually too narrow for the construction of long sills, however, and it is easier to install protective works directly on the channel side slopes.

Loose rock was applied directly for bank protection. In all cases, the design provided for excavation of the side slopes to a depth of about 1 foot. The objective of this excavation was to permit placement of the rock flush with the surrounding side slope surface to increase stability of the protection (fig. 15). Excavation of surface materials also assured that the rock would not be set on vegetation that could weaken the bank protection.

Where the gully side slopes were steeper than 1.25 to 1.00, a wire mesh fence, secured to steel posts, was designed approximately flush with the original slope surface, and the space behind the fence filled with loose rock (fig. 16). The fence added strength to the protective works and prevented displacement of the rock.

Figure 15.--The rod, 5.5 feet long, is placed on apron of this double-fence rock check dam. Note the rock for gully side-slope protection is embedded in the soil without reinforcement by a fence.

The height of the bank protection depends on the characteristics of channel, flow, and structure. Where gullies have wide bottoms, and spillways are designed to shed the water only on the channel floor, the required height will depend on the depth of the tailwater expected on the apron. But where the waterfall from a check dam will strike against the gully banks, bank protection must be of sufficient height to prevent the water from splashing against unprotected bank areas. Otherwise, new erosion may be initiated.

In gullies with V-shaped cross sections, the height of the bank protection was made equal to the elevation of the upper edges of the freeboards of the structures. In general, with increasing distance from the dam, the height of the bank protection decreased.

Construction

Motorized equipment was utilized to the fullest extent possible in the construction of the check dams. Hand labor was used only for work that could not be satisfactorily performed by machinery. In general, besides the equipment and its operators, not more than two laborers were employed in the construction. For most operations, conventional types of





Figure 16.--
At this double-fence rock check dam, steepness of the side slopes required that the rock side-slope protection be stabilized with a fence.

equipment were used: backhoe, clam shell, crawler-type tractor with blade, air compressor, dump truck, and bucket loader. Since conventional post-driving equipment, such as a pile driver, is not suitable for the construction of check dams, an attachment to a pavement breaker was designed (Heede 1964). This breaker, driven by an air compressor, was easy to handle in the gullies and drove steel posts efficiently into the dense, cohesive soils.

Before construction started, the following design features were staked and flagged conspicuously:

1. The centerline of the dam or the key trenches, respectively, was marked on each bank. The stakes were set away from the gully edge to protect them during construction.
2. The crest of the spillway was designated by a temporary bench mark in the gully side slope sufficiently close to be of value for the installation of the dam.
3. The downstream end of the apron was marked.
4. For loose-rock and wire-bound dams, the upstream and downstream toes of the dam proper were flagged.

This amount of staking greatly facilitated dam construction. Caution was required during excavation, however, to avoid destroying the stakes before the main work of installation began.

The construction of all dams started with the excavation for the structural key, the apron, and the bank protection. This very important work was performed by a backhoe, which also cleaned vegetation and loose material from the site.

Since all trenches were designed to have a width of 2 feet, a 1.5-foot-wide bucket was used on the backhoe. Two types of backhoes were used. One was mounted on a rubber-wheeled vehicle and operated from a turntable, which permitted the backhoe to rotate 360 degrees. This machine traveled rapidly between locations where the ground surfaces were not rough, and worked very efficiently in gullies whose side slopes and bottoms could be excavated from one or both channel banks. The other type of backhoe was attached to a crawler tractor. This type proved to be advantageous at gullies that were difficult to reach, and with widths and depths so large that the backhoe had to descend into the channel to excavate. In deep gullies with V-shaped cross sections, temporary benches on the side slopes were necessary. Often, the bench was constructed by the tractor with blade before the backhoe arrived.

After the excavation was finished, trench and apron were filled by dumping rock from dump trucks. Where gully cross sections were so large that dumping did not completely fill them, the backhoe was used to finish the job.

The dump trucks were loaded by a bucket loader at the stockpile on the watershed. The bucket scooped up a certain amount of soil along with the rock, which later was dumped with the rock into the structures. Since soil is undesirable in a rock check dam because of the danger of washouts by future flows, a bucket with a grilled bottom would have been more suited. Most of the soil could have been removed from such a bucket by shaking it before the trucks were loaded. Other devices such as a grilled loading chute would also have been applicable.

If loose-rock check dams were to be constructed, the dumping operations continued after any large openings in the trench or apron had been filled with rock. For structural stability, as much volume of rock was piled up by dumping as possible. This assured a greater density in the structures, and dam sites were closer to the angle of rest of the rock than if the rock were piled by hand. Gully geometry sometimes restricted the dumping of rock, such as at deep channels where the direction of rock fall could not be controlled, or at wide channel sections where the dumped material would not reach the center of the gully. In these cases, the rock was dumped on the gully bank and the backhoe placed the rock in the structure. In gully reaches more than 20 feet deep, the backhoe was not practical and a clam shell was used (fig. 17) on a 42-foot boom attached to a turntable. The machine

also placed the rock for the bank protection. Hand labor was required only for the last finish work on the structures.

Special attention was needed at the spillway and freeboard. In loose-rock and wire-bound structures, where the shape of the dams is not outlined by a fence as in the other types, experience showed there is a tendency to construct the spillways smaller than designed.

A commercial, galvanized stock fence, 4 feet wide was used in the construction of dams that required the use of wire mesh. The stay and line wires were of 12-1/2-gage low-carbon steel, the top and bottom wires of 10-gage low-carbon steel, and the openings in the mesh were 6 inches. To connect ends of the fence or to attach the fence to steel posts, a galvanized 12-1/2 gage coil wire was used.

For wire-bound dams, the wire mesh of required length and width was placed on the gully bottom and side slopes after the trench and apron had been filled with rock (fig. 18). The required dimensions of the wire mesh were taken from the construction plans. Generally, several widths of mesh were needed to cover a structure from bank to bank. If several sheets of the fence material were required, the sheets were wired together with coil wire where they would be covered with rocks. The parts to be unburied were left unattached to facilitate the fence-stringing operations around the structure.

Figure 17.--A clam shell empties rock into a double-fence structure. Note the reinforcing fence for the gully side-slope protection.





Figure 18.--

The trench across the gully, representing the key of the planned wire-bound check dam, and the apron are filled with rock. The wire-mesh mat is placed on the fills. After this mat has been temporarily fastened to the gully side slopes, the construction of the dam proper can start.

Before the rock could be placed on the wire mesh for the installation of the dam proper, the mesh had to be temporarily attached to the gully banks. Otherwise, the wire mesh, lying on the gully side slopes, would be pushed into the gully bottom by the falling rock and buried. Usually, stakes were used to hold the wire mesh on the banks.

After the dam proper had been placed and its final shape attained, the fence was bound around the structure. Fence stretchers were applied to pull the upstream ends of the fence material down tightly over the downstream ends, where they were fastened together with coil wire. Then the bank protection below the dam was installed.

The installation of single- and double-fence dams began with the construction of the fences after excavation was completed (fig. 19). Construction drawings had to be followed closely here because the final shapes of the dams were determined by the fences. Conventional steel fenceposts from 5.5 to 10 feet long were used. In some locations, the great height of posts offered difficulties for the operator of the driving equipment. Where gully bottoms were wide and accessible, the powerwagon that carried the air compressor, air hoses, pavement breaker, and fence materials was driven to the construction site and served as scaffold for the operator. When the gully was inaccessible, scaffolds were improvised.

At single-fence dams, dumping of rock was practical if the gully was not excessively deep or wide. At double-fence structures, a backhoe or clam shell was always required. The rock was placed in layers, and each layer was inspected for large voids by a man on the ground. The large voids were closed manually by rearranging rocks.

Much time and effort can be saved during construction if a realistic equipment plan is established beforehand. Such a plan requires an intimate knowledge of the cross-sectional dimensions of the gullies and their accessibility to motorized equipment. The amount of pioneer roads that will be needed because of lack of access is not only important for equipment considerations, but will also enter into the cost of the construction.

As a general rule, it appeared to be advantageous to use heavier and larger sized equipment if its mobility was adequate. Although hourly costs for heavier machines are usually greater, the total costs for a job are reduced if larger operations are involved.

With a few exceptions, conventional construction equipment is not sufficiently mobile to operate in rough topography without pioneer roads. In watershed rehabilitation projects such as gully control, roads are undesirable because any road construction disturbs the ground surface and may lead to new erosion.

Limited mobility of the machinery originated from one or all of the following factors: (1) insufficiently powered engines, (2) large total weight of the machinery, and (3) mounting of the equipment on rubber wheels. A good example was the backhoe. If a crawler-type backhoe with a powerful engine was used, the total weight of the equipment was so large that it could not climb slopes steeper than 20 percent. If a crawler-type backhoe with lesser weight was used, the engine lacked the power for unlimited operations in rough topography, and pioneer roads were needed.

Costs

During construction, detailed records were kept on time and materials expended for each work phase and structure. The absolute values of our costs, based on these records, may be unrealistic for conditions different from those of our watershed. But the indices of these values give an insight into the intricate relationships between costs of different types of check dams and between costs of dams and channel characteristics. These indices will hold true also for other watersheds.

Table 1 shows the expenditures of materials, work time, and funds required for the placement of 1 cubic yard of structure. These data should be regarded as approximations for the following reasons:

1. Sizes of samples varied greatly between the types because changes in the original design were required during construction.
2. The data for all dams within one type were averaged, regardless of height of dam, gradient, or width of gullies.
3. Dam heights ranged from 1.5 to 7 feet, but were not evenly distributed within or between the types. Most of the dams had an effective height from 2 to 5 feet.

Therefore, in evaluating the cost per cubic yard of structure, emphasis should be placed on the cost relations between the different types of dams. The cost per cubic yard is smallest for check dams built from loose rock, and greatest for a double-fence structure. The reason for this relationship is that all dams with the exception of the loose-rock



Figure 19.--Installing fences for a double-fence rock check dam.

type require additional materials such as wire mesh and steel posts. The relations are expressed as cost indices in the table. For example, on the average, 1 cubic yard of a loose-rock structure costs 32 percent less than 1 cubic yard of double-fence type.

In accordance with general experience in economics, our data demonstrate that the cost of 1 cubic yard of structure decreases with increased effective height:

Type of check dam:	Effective height of dam	
	(2 feet)	(6 feet)
Loose rock	\$12.81	\$11.18
Single fence	17.59	14.99
Double fence	17.59	15.65

Check dams were grouped by effective height in table 2. For each group, five gully sites were randomly selected and three different types of dams were designed for each

Table 1. --Expenditures for the placement of 1 cubic yard of loose-rock check dam

Type of check dam, number of dams, and cost per unit	Materials					Machine time					Hand labor	Total cost	Cost index
	Rock	Wire mesh	Coil wire	Re-bar	Steel posts	Air compressor	Backhoe	Clam shell	Loader and dump truck	D-7 tractor			
	Cu. yd.	Linear ft.		Number				Hours					
Double-fence: 40 dams	1	6	3	1.6	1.2	0.16	0.16	0.06	0.25	0	0.57	\$16.69	1.00
Single-fence: 12 dams	1	4	2.4	2.3	.8	.14	.12	0	.25	0	.51	14.59	.87
Wire-bound: 7 dams	1	6.4	.6	0	0	0	.05	0	.25	0	.48	12.16	.73
Loose rock only: 53 dams	1	0	0	0	0	0	.09	0	.25	.02	.02	11.30	.68
	<div>Dollars</div>												
Cost per unit of measure	8.03	0.083	0.004	0.07	1.00	7.00	11.00	12.00	8.00	12.00	2.17		

Table 2. --Development of rock volumes and installation costs of different types of check dams with different effective heights

Effective height of dam (Feet)	Type of dam	Relative rock volume	Cost index ¹ per cubic yard	Relative cost of dam
		Percent		Percent
2	Double-fence	100	1.00	100
	Single-fence	120	.87	104
	Loose rock only	186	.68	126
3	Double-fence	100	1.00	100
	Single-fence	144	.87	125
	Loose rock only	230	.68	156
4	Double-fence	100	1.00	100
	Single-fence	150	.87	131
	Loose rock only	262	.68	178
5	Double-fence	100	1.00	100
	Single-fence	164	.87	143
	Loose rock only	273	.68	186

¹ Based on average costs shown in table 1.

site. Rock volumes for each type were compiled and, based on the volume requirements of the double-fence structure, the relative volumes of the dams were computed. When these relative figures were multiplied by the respective costs of placement, the relative costs of the dam types by effective height were obtained. The data demonstrate that double-fence dams require less rock and are thus cheaper to install than the other types. This advantage increases with increasing height of structure and cost of rock.

The above cost relationships hold true if total gully treatments by the different types of dams are compared. In table 3, rock vol-

umes and costs required for the treatment of two gullies with gradients of 5 and 15 percent, respectively, are compiled. Loose-rock, single-fence, and double-fence structures with effective heights of 2 and 6 feet are considered. The data on the total cost of gully treatments show that, regardless of the gully gradient, treatment with double-fence dams is cheaper. It also follows that cost of gully control increases with increasing gully gradient between 5 and 15 percent.

Table 3 illustrates further that reducing the total number of check dams in a gully by increasing the effective height of the structures usually does not save installation costs.

Table 3. --Installation costs of loose-rock check dams, by gully gradients and effective structural heights

Type of check dam and gully gradient (Percent)	Effective height of dam	Total dams required	Volume of rock--		Installation cost--		Cost ratio
			in one dam	Total	for 1 cubic yard	Total	
	Feet	Number	Cubic yards				
Double-fence:							
15 percent	2	10	6.5	65.0	\$17.59	\$1,143.35	0.84
	6	4	15.4	61.6	15.65	964.04	
5 percent	2	2	6.4	12.8	17.59	225.15	1.07
	6	1	15.4	15.4	15.65	241.01	
Single-fence:							
15 percent	2	10	7.5	75.0	17.59	1,319.25	1.24
	6	4	27.2	108.8	14.99	1,630.91	
5 percent	2	2	8.1	16.2	17.59	284.96	1.56
	6	1	29.6	29.6	14.99	443.70	
Loose rock only:							
15 percent	2	10	12.1	121.0	12.81	1,550.00	1.61
	6	4	55.9	223.6	11.18	2,499.85	
5 percent	2	2	12.0	24.0	12.81	307.44	1.99
	6	1	54.7	54.7	11.18	611.55	

Savings can be achieved only if higher double-fence dams are installed on a gradient steeper than 5 percent.

The cost ratios show further that a treatment by 6-foot dams may cost as much as 100 percent more than by 2-foot dams. With increasing gully gradient, the cost ratios decrease.

The cost relationship between the treatments by different types of structures with different effective heights can be explained by the geometry of the dams. Gully gradient and height of structure influence this geometry, which in turn determines the volume. Thus, rock volumes in loose-rock dams increase much more with height than those of single-fence or double-fence types. Due to the structural geometry also, volumes of single-fence types of a given height decrease with increasing gully gradient, while volumes at double-fence dams do not change.

Results and Recommendations

1. A good-quality, well-graded rock should be used in rock check dams. Most of the rocks should be sufficiently large and heavy to resist the impact of the flows.
2. The rock used for filling the keys of the check dams should be smaller (80 percent smaller than 3 inches).
3. Although the general locations of check dams will be determined by spacing re-

quirements, the final structural sites should be selected on the basis of gully characteristics that benefit construction and costs.

4. In gullies with cohesive soils, keys with depth and width of 2 feet were sufficient.
5. For adequate protection of gully bottoms and side slopes, the length of apron and side slope rock-work should be 1.5 times the dam height where the gully gradient is 3 to 15 percent, and 1.75 times dam height for steeper gully gradients.
6. Spillways of check dams, installed on forested or brushland watersheds, should be designed to accommodate not only the expected peak flow, but also the debris in the flow. In general, such a design will result in requirements for discharge capacities much larger than those derived from peak flow calculations only.
7. Cross sections of spillways should be trapezoidal. Discharge capacity increases rapidly with depth of flow over the spillway, and debris can be more easily carried through the spillway.
8. If the spillway is wider than the gully bottom, the channel side slopes below a rock check dam shall be protected to the crest of the free boards of the structures.
9. Where tail waters do not occur below a check dam, a low end sill should be installed at the apron to force the ground roller, developed by the hydraulic jump, away from the apron.
10. Types of conventional construction equipment, as often used by land managers, are

suitable for the installation of rock check dams.

11. The amount of hand labor required in the construction of different types of rock check dams is small, if related to the total cost. Thus, 2 to 8 percent of the total cost of placement for 1 cubic yard of structure was expended for manual work, depending on the type of dam.
12. The cost for the installation of 1 cubic yard of rock check dam decreases with increasing effective height of structure.
13. Double-fence rock check dams require less rock volume and are less expensive than the other types of rock check dams of equal effective height.
14. Where double-fence rock dams cannot be used, cost will be reduced by using low dams on gully gradients less than 15 percent.
15. Where double-fence dams can be used, a treatment by higher structures is less expensive on gradients steeper than 5 percent.

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